# Estimated seismic performance of a standard NZS3101:2006 RC office building during the 22 February 2011 Christchurch earthquake

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**ABSTRACT:** This paper discusses the seismic performance of the standard RC office building in Christchurch that is given as a structural design example in NZS3101, the concrete structures seismic standard in New Zealand. Firstly the push-over analysis was carried out to evaluate the lateral load carrying capacity of the RC building and then to compare that carrying capacity with the Japanese standard law. The estimated figures showed that the carrying capacity of the New Zealand standard RC office building of NZS3101:2006 was about one third of Japanese demanded carrying capacity. Secondly, time history analysis of the multi-mass system was performed to estimate the maximum response story drift angle using recorded ground motions. Finally, a three-dimensional analysis was carried out to estimate the response of the building to the 22nd February, 2011 Canterbury earthquake. The following outcomes were obtained. 1) The fundamental period of the example RC building is more than twice that of Japanese simplified calculation, 2) The example building's maximum storey drift angle reached 2.5% under the recorded ground motions. The main purpose of this work is to provide background information of seismic design practice for the reconstruction of Christchurch.

## 1 INTRODUCTION

The Christchurch earthquake occurred on 22 February 2011. Unfortunately, the epicentre was close to Christchurch central business district (CBD), where the distance was approximately 10km south-east of the CBD. The M6.3 earthquake caused great damage to many buildings in Christchurch CBD. Besides many collapsed masonry historical buildings, two reinforced concrete (RC) buildings collapsed and several others were severely damaged. To investigate the cause of such disaster, details of the damaged buildings are needed. As a result, to understand the New Zealand seismic design, the design example of NZS3101:2006 (NZS, 2006), which was published by NZCS (NZCS, 2008), was picked up as one of the ordinary RC buildings in Christchurch. The example was a 10 storey RC building for office use as shown in Fig-1. Although the building is only a hypothetical example, this example was considered as a bible for many engineers. This paper treated this example as a general RC building and took the zone factor in Christchurch equal to 0.22, although the zone factor was increased to 0.30 and formally ratified in November 2011. The main purpose of this paper was to provide a comparison between the seismic design methods in Japan

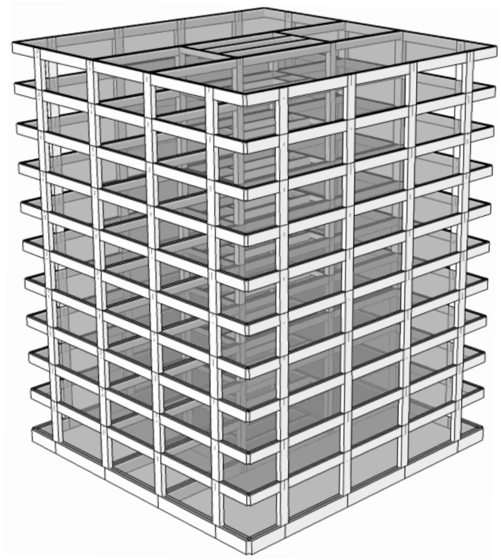


**Fig-1 The example of NZS3101 RC buildings in Christchurch (NZCS, 2008)**

and New Zealand. In Japan, several earthquake disasters have occurred and many precious lessons have been learnt from experience, taking technical facts and social demand in Japan at that time into account. The authors, herein, present some technical considerations using Japanese methods and some lessons for seismic design learnt throughout this study, as well as the post-earthquake investigation carried out by the first author as a member of the Joint mission of Japan Association for Earthquake Engineering and Architectural Institute of Japan to the areas affected by the Feb. 22, 2011, Christchurch earthquake from 9th March and 14th March. The fact-finding mission was conducted in conjunction with Professor. Y. Nakano of The Tokyo University and Associate Professor. S. Kono of Kyoto University from 9th March and 14th March, 2011.

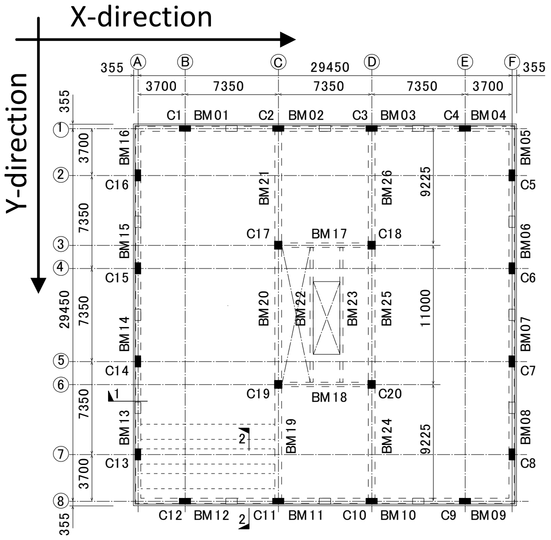



**Fig-2 Building plan (NZCS, 2008)**

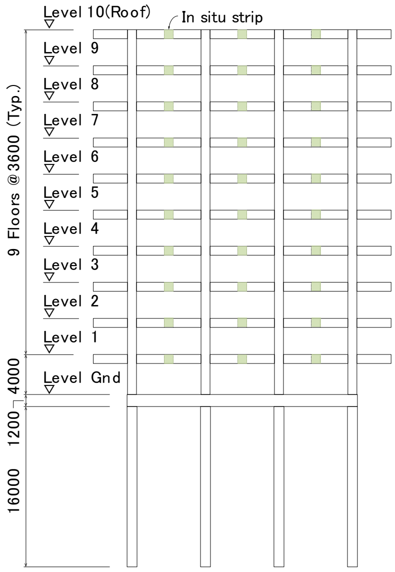
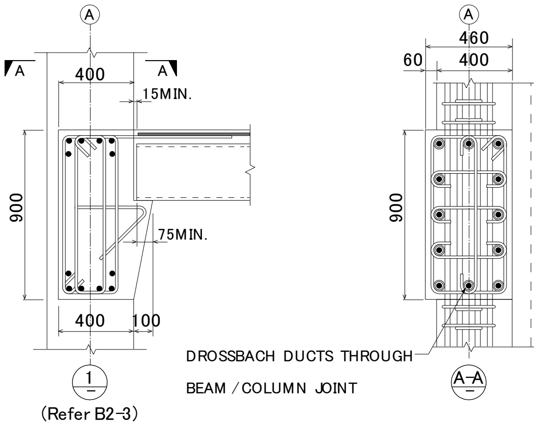



**Fig-3 Exterior frame elevation(NZCS, 2008)**

**Table-1 Section size (NZCS, 2008)**

| SUMMARY OF MEMBER SIZES | |
|---|---|
| ***COLUMNS ALL LEVELS*** | |
| C1-C16 | 900 x 460 |
| C17-C20 | 650 x 600 |
| ***BEAMS – GROUND*** | 1,200 x 600 |
| ***BEAMS – LEVELS 1-10*** | |
| BM01-BM04, BM09-BM12 | 900 x 400 |
| BM05-BM08, BM13-BM16 | 900 x 400 |
| BM17, BM18 | 550 x 350 |
| BM19-BM21, BM24-BM26 | 750 x 530 |
| BM22, BM23 | 750 x 250 |
| ***PILES*** | 1,000 DIA. |
| ***FLOORING*** | |
| GROUND | 150 INSITU ON GRADE |
| LEVELS 1-10 | AS NOTED |




Main Beams
Concrete : 30MPa, Steel bars : Grade300
Ratio of tension steel bars : pt=0.89%
Ratio of shear reinforcement : pw=0.33%
(Exterior Columns)
Concrete : 30MPa, Steel bars : Grade300
Ratio of Longitudinal bars : pg=1.47%
Ratio of shear reinforcement : pw=0.57%

**Fig-4 Beam, Column sections (NZCS, 2008)**

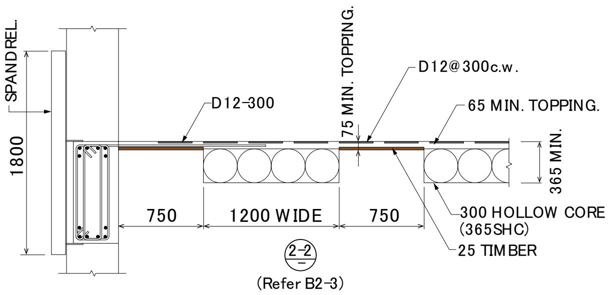



**Fig-5 Floor sections (NZCS, 2008)**

## 2 OUTLINE OF THE EXAMPLE RC 10 STOREY BUILDING

The examined design example is a 10 storey reinforced concrete building for office use in Christchurch. The design of the RC building was published by Cement & Concrete Association of New Zealand (NZCS, 2008). Fig-2 and Fig-3 show the typical floor plan and elevation, respectively. The typical floor is 29.5 meters square. The building total height is 36.4 meters. While the 1st floor height is 4.0 meters, other typical floor-to-floor height is 3.6 meters. Interior beams are provided in only one direction, and are not designed for earthquake loading. These beams support hollow core slabs. Exterior frames are composed of columns with a section of 460 mm by 900 mm and beams with a section of 400 mm by 900 mm and a regular span of 7.35 meters, as shown in Fig-4. Most of the earthquake loading is resisted by these exterior frames. The frame elements' section sizes are shown in Table-1. The integrated precast beams and joint was applied to the building. The center of each exterior beam is casting concrete. This construction method is also used in Japan. Appropriate amounts of shear reinforcement, similar to that practiced in Japan, are provided in the beams, as shown in Fig-4. It is particularly pointed out that the flexible detail provided between beams and parallel spanning hollow core slab as shown Fig-5, to minimise differential deflection, is not applied in Japan.

## 3 BASIC SEISMIC PERFORMANCE OF THE EXAMPLE RC 10 STOREY BUILDING

### 3.1 Estimation of earthquake loading for the RC 10 storey building in Christchurch

The conditions set to evaluate the earthquake loading for the example building was as follows (NZCS, 2008). The building was composed of ordinary RC ductile frames based on NZS1170.5 before the Christchurch earthquake in 2011. The ductility factor $\mu$ =4.0 was selected as the standard value of moment frame structures. The zone factor 0.22 was chosen for Christchurch. The return period was assumed 500 years. The shortest distance from Christchurch to the nearest fault was considered more than 100 kilometres. The main equations are given below.

*Equivalent Static Forces: NZCS 2008 for 10 storey RC office in Christchurch (before 2011)*

*Period Estimate* $T_1=0.11h_n^{3/4}=0.11 \times 36.4^{3/4}=1.63(s)$ (1)

*Lateral force coefficient* $C(T)=C_h(T) \times Z \times R \times N(T,D) =0.823 \times 0.22 \times 1.0 \times 1.0=0.181$ (2)

*Horizontal design action coefficient for Ultimate design stage*

$$C_d(T_1)= C(T_1)S_p/k_\mu=0.181 \times 0.7/4.0 =0.032 \geqq (Z/20+0.02)R_u \geqq 0.03 R_u \quad (3)$$

*Where* $C_h(T)$*; 0.823-Linear interpolation , R ; Return period factor* $R=1-1/500 \fallingdotseq 1.0$

*N(T,D) ; Near fault factor,* $S_p$ *;* $=1.3-\mu$ *the structural performance of 4.4 NZS1170.5*

$k_\mu$*; for soil class C (Shallow soil site),* $T_1 \geqq 0.7(s)$*,* $k_\mu= \mu$

Fig-6 shows the calculated lateral force coefficient related to the natural period, which is an important factor in the calculation. The calculated period by NZS1170.5 was about twice that of the Japanese standard law, which suggests that the natural period is equal to 0.02 times the total height of buildings. The evaluation of the natural period by NZS1170.5 is, actually, based on UBC91. The method is similar to the eigenvalue analysis. The calculated natural period was 1.55 and 1.41 second for X and Y directions, respectively. Such big difference between UBC91 method and Japanese standard law was explained by the number of beams. The beams influenced the natural period and stiffness of the building. The horizontal design action coefficient for the ultimate design stage is shown in Fig-6. The blue circle shows the calculation results for the ductility factor

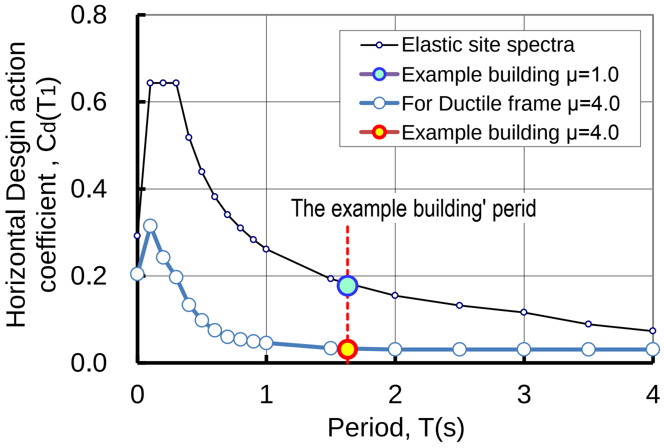



Fig-6 Floor sections (NZCS, 2008)

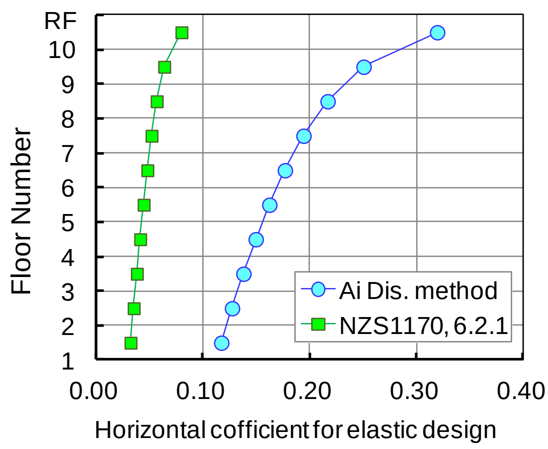



Fig-7 Distribution of storey shear coefficient

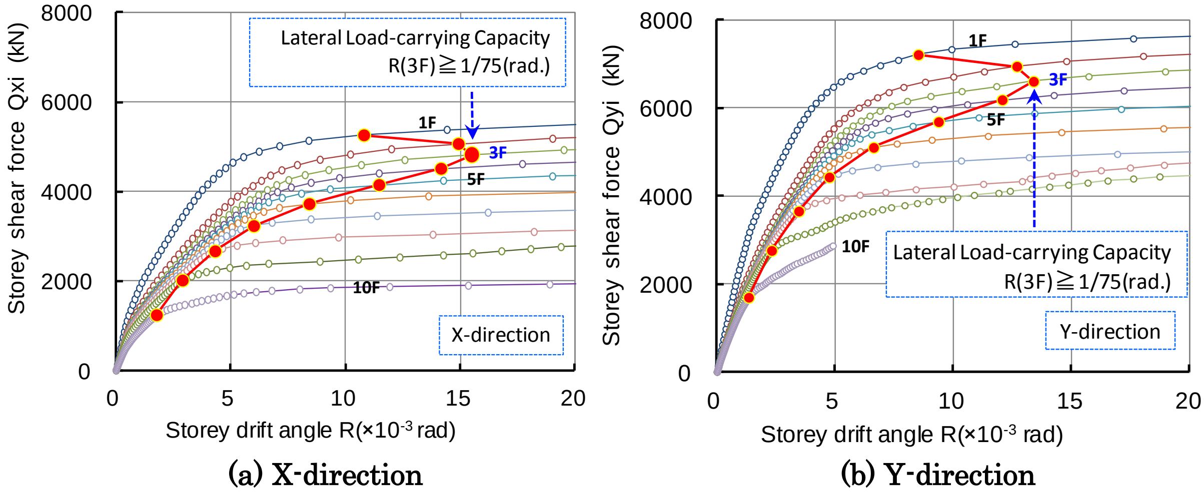



Fig-8 Push-over analysis results (Storey shear force-storey drift angle relationship)

$\mu$ =1.0. The red circle shows the lateral force coefficient for the case $\mu$ =4.0. This calculation was based on the principal of displacement conservation. The result for $\mu$ =4.0 is one-fourth of the result for $\mu$ =1.0. This shows the importance of the ductility factor in the evaluation of earthquake loading.

### 3.2 Distribution of story shear coefficient

To carry out the push-over analysis and Multi-mass earthquake response analysis, setting the distribution of storey shear coefficients was necessary. The distribution evaluation is needed to represent dynamic loads by an equivalent static loading. Ai distribution method was proposed to consider the influence of higher vibration under earthquake loading. The method was enforced by THE BUILDING STANDARD LAW in 1981(BCJ, 2011). The Ai distribution method was proposed in Japan for buildings of long-period with height less than about 60 meter. The NZS1170.5 method is similar to the Japanese one. The Ai distribution of the example building is presented in Fig-7. The calculation considered the standard base shear coefficient equal to 0.20 for an elastic behaviour based on the allowable stress design. The allowable stress design assumes that a building experiences a moderate earthquake a few times in its life period. In Fig-7, the green squares show the earthquake equivalent static loading coefficient for the example building of NZS1170.5. A large difference relative to earthquake load conditions appears between Christchurch before 2010 and Japan.

### 3.3 Push-over analysis of RC 10 storey building in Christchurch

To estimate the potential lateral load carrying capacity of the example building, push over analyses of the building were carried out. The results for X-direction and Y-direction are shown in Fig-8. The carrying capacity was assumed to be reached when the maximum story drift angle reached 1/75 rad.

**Table-2 Lateral load carrying capacity of the example building**

| | X-direction | | | Y-direction | | |
|---|---|---|---|---|---|---|
| Storey | Demanded Lateral Load-carrying Capacity Qun (kN) | Lateral Load-carrying Capacity Qu (kN) | Qu/Qun | Demanded Lateral Load-carrying Capacity Qun (kN) | Lateral Load-carrying Capacity Qu (kN) | Qu/Qun |
| 10 | 4301 | 1233 | 0.287 | 4301 | 1692 | 0.393 |
| 9 | 7009 | 2009 | 0.287 | 7009 | 2757 | 0.393 |
| 8 | 9283 | 2661 | 0.287 | 9283 | 3651 | 0.393 |
| 7 | 11253 | 3226 | 0.287 | 11253 | 4426 | 0.393 |
| 6 | 12968 | 3718 | 0.287 | 12968 | 5101 | 0.393 |
| 5 | 14454 | 4143 | 0.287 | 14454 | 5685 | 0.393 |
| 4 | 15725 | 4508 | 0.287 | 15725 | 6185 | 0.393 |
| 3 | 16790 | 4813 | 0.287 | 16790 | 6604 | 0.393 |
| 2 | 17657 | 5062 | 0.287 | 17657 | 6945 | 0.393 |
| 1 | 18334 | 5256 | 0.287 | 27501 | 7211 | 0.262 |

**Table-3 Previous observed earthquakes**

| | El-Centro NS | Taft EW | Hachinohe NS |
|---|---|---|---|
| Level 1 25cm/s | 255.4 $m/s^2$ | 248.4 $m/s^2$ | 165.1 $m/s^2$ |
| Level 2 50cm/s | 510.8 $m/s^2$ | 496.7 $m/s^2$ | 330.1 $m/s^2$ |
| Leve1/ Earthquake that occurred rarely | | | |
| Level2 / Earthquake that occurred extremely rarely | | | |

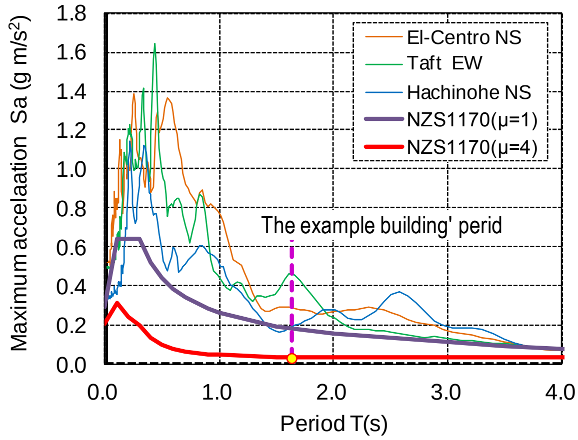
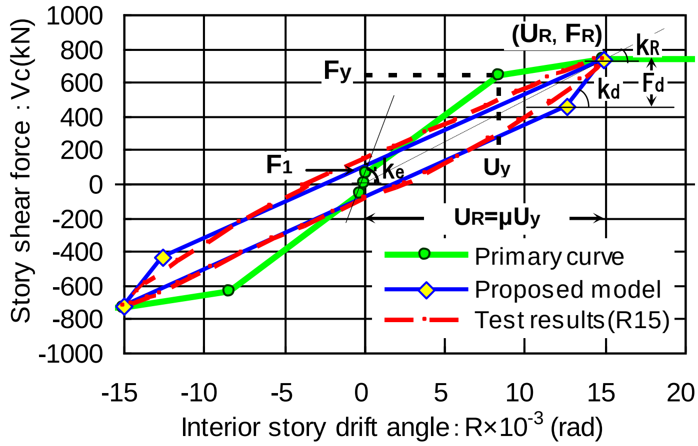



**Fig-9 Acceleration spectra of previous observed earthquake**




Unloadingbreakingload

( the value from each peak load)

$$F_d = \left(\frac{F_y - F_1}{2}\right)\mu + \left(\frac{3F_1}{2} - \frac{F_y}{2}\right), F_d \leqq F_R, 1 \leqq \mu \leqq 3$$

Firstunloadingstiffess

$$k_d = \frac{k_e + k_R}{2\sqrt{\mu}} \quad ,1 \leqq \mu$$

$(U_1, F_1)$ : Firstbreakingpoint (limitedelasticpoint)

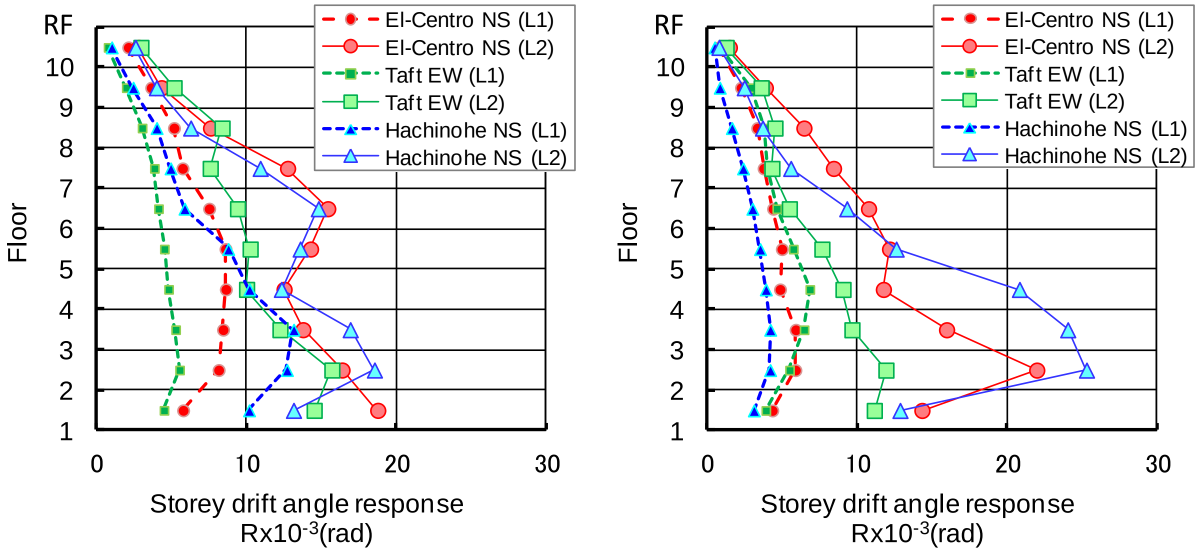
$(U_y, F_y)$ : Secodbreakingpoint (flexural yielding point)

$\mu = U_R/U_y$ : ductility factor

$(U_R, F_R)$ : Each Peak points on primary curve

**Fig-10 Outline of the hysteresis model (Ishikawa, 2007)**

Table-2 lists the results as well as a comparison between the lateral load-carrying capacity and the demanded one based on Japanese standard law. Japanese standard law demands that base shear coefficient be more than 0.30 for RC ductile frames. The load-carrying capacities of the example building in X and Y-directions are about 0.30 and 0.40 times the demanded ones, respectively. It is clear that Japanese earthquake condition is not equal to New Zealand one. The different results between New Zealand and Japan are due to the difference in the assumed values of the ductility factor. In Japan, the value relative to RC frames is about 2.0 for the calculation of load-carrying capacity.




**(a) X-direction** **(b) Y-direction**

**Fig-11 Multi-mass earthquake response analysis results**

### 3.4 Multi-mass earthquake response analysis

Firstly, the time history analysis using a multi-mass model was performed using previous recorded ground motions. The multi-mass model was set up using the push-over analysis results of the preceding section. Table-3 shows the characteristics of the ground motions. Level-1 and Level-2 earthquakes were calibrated to the maximum velocity of 25 m/s and 50 m/s, respectively. Fig-9 shows the results of the elastic acceleration response spectra using 5% damping factor. Nevertheless, the earthquake response analysis used 3% damping for the example RC building. The example building's natural period is indicated in the figure. Fig-10 shows the adopted hysteresis model for the analysis. The quadri-linear model was established using 10 beam-column joints data (Ishikawa, 2007). Fig-11 shows the results in X and Y-directions. The maximum response story drift angle in X-direction under Level-1 and Level-2 earthquake waves were 1/147 rad and 1/53 rad, respectively. Similarly, in Y-direction the results under Level-1 and Level-2 ones were 1/76 rad and 1/39 rad, respectively. These values corresponded to twice the Japanese criterion.

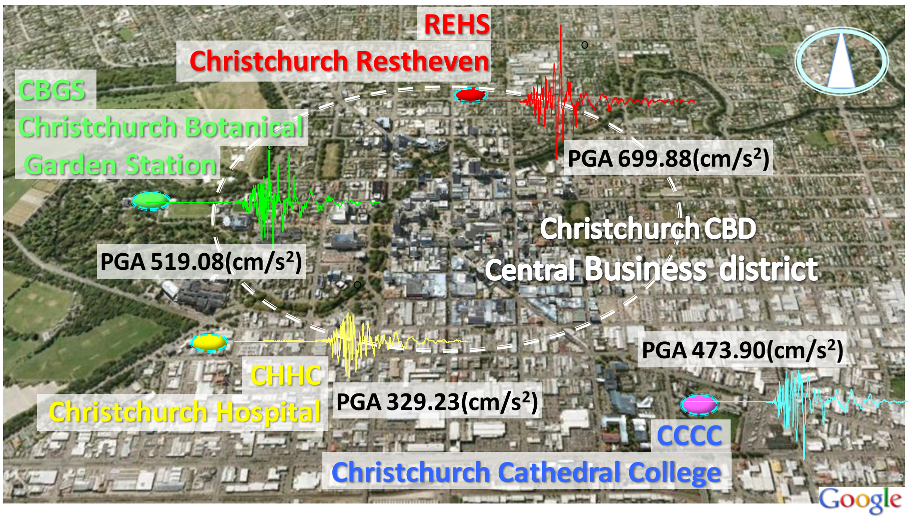
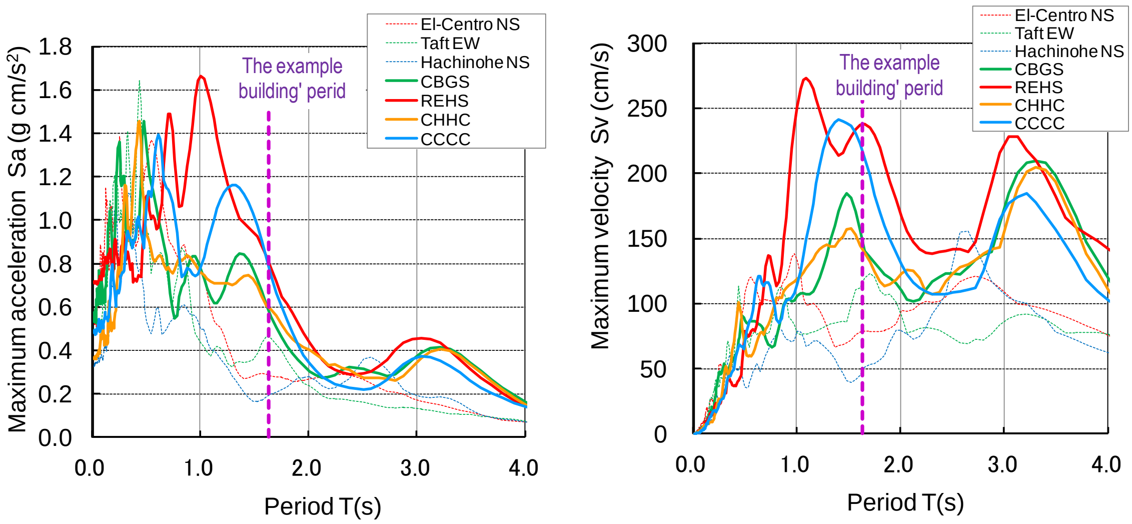



**Fig-12 Four seismic observation points in 22nd February, 2011, refer to (Brendon, 2011)**




**(a)Acceleration spectra** **(b) Velocity spectra**

**Fig-13 Four seismic observation points in 22nd February, 2011**

## 4 SEISMIC PERFORMANCE UNDER OBSERVED EARTHQUAKE ON 22ND FEB. 2011

### 4.1 Observed earthquake on 22nd February, 2011

Fig-12 shows the locations of four seismic observation points around Christchurch CBD. The maximum recorded acceleration was 699.9 cm/s$^2$ at REHS. Fig-13 shows the spectra of acceleration and velocity using 5% damping factor. The maximum acceleration spectrum was 1.67G around 1.0 second period which corresponds to 18.9 meter-high RC buildings using NZS1170.5. As to velocity spectra, recorded waves were larger than previous earthquakes. The velocity spectra indicate a second peak around 3.0 sec. CBGS indicates cyclic mobility by liquefaction. It is important to remind here that Christchurch was a swamp area until 18 Century.

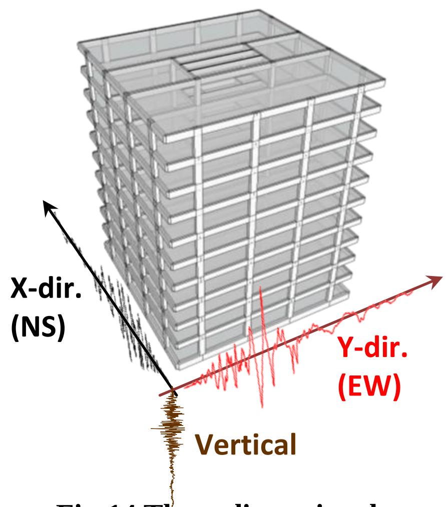
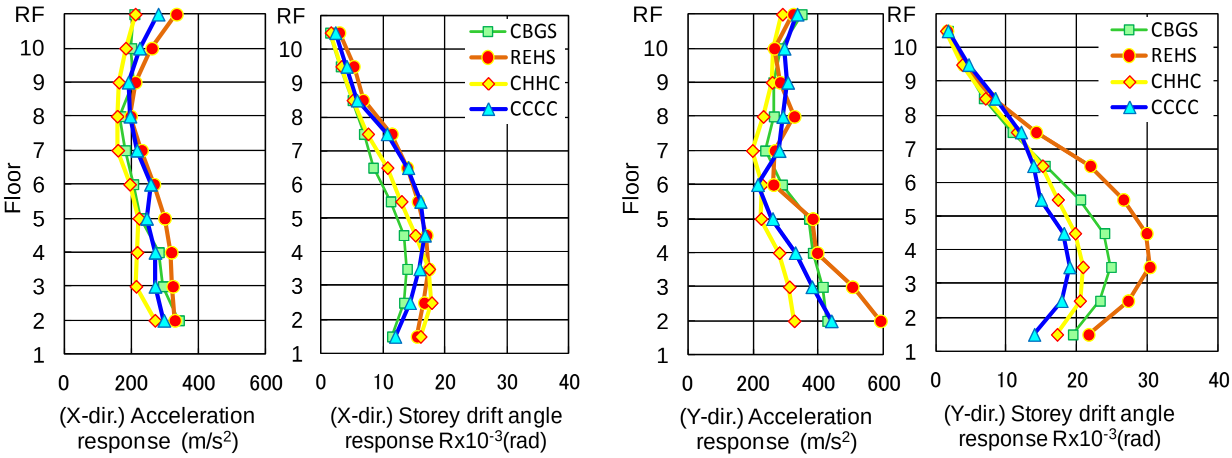



**Fig-14 Three dimensional analysis model**

### 4.2 Time history three-dimensional analysis

Time history three-dimensional analysis was carried out to estimate the response of the example building to the 22nd February, 2011 Canterbury earthquake. In this analysis, three-dimensional ground motions were simultaneous inputted. X and Y directions were assumed North-South and East-West directions, respectively. The Fig-15 shows the results of the three-dimensional time history analysis. Fig-15 shows the acceleration response in X and Y directions using the acceleration response and the storey drift angle response in X and Y directions. The analysis assumed a fixed foundation. Because the frames in X-direction had no interior beams, then the frame story stiffness in X-direction was lower than in Y-direction. Nevertheless, the story drift angle response in X-direction is relatively smaller than in Y-direction. Because the east-west direction ground motion is larger than the north-south ground motion on 22$^{nd}$ February at around Christchurch CBD. It's striking that maximum storey drift angle of X and Y-directions reached 1/56 rad. and 1/33 rad., respectively. Ductility factors in X and Y directions were 3.83 and 6.88, respectively. These values mean that actual columns, beams and joints need more reinforcement to achieve higher ductility. Furthermore, under such response conditions, all finishing's including glazing would fall down resulting in danger and potentially fatal injuries. In Japan, design principal criteria are less than 1/100 rad. for storey drift angle and 2.0 for ductility factor under Level-2 earthquake as shown in Table-3. In other words, the recorded earthquake affected the buildings in Christchurch greater than what would be expected.




**(a)X / North-South-direction** **(b) Y / East-West-direction**

**Fig-15 Time history three-dimensional analysis results**

## 5 SUMMARY

In this paper, the seismic performance of the NZS3101 example RC building was investigated using push-over and time history analyses. The following conclusions can be drawn.

1) The example building's natural period was equal to twice that of Japanese simplified method. Main reason was in the difference of number of beams. Load carrying capacity of the example building was equal to one-third of the design criteria based on Japanese standard law

2) The multi-degree time history analysis showed that the example building's maximum story drift angel reached 1/76 and 1/40 rad. under the normalized previous recorded ground motions using velocity 25 cm/s and 50 cm/s, respectively for rare and extremely rare earthquakes.

3) During three-dimensional time history analysis, the maximum story drift angle of the NZS3101 example building reached about 1/30 rad.

4) For large deformations under severe earthquakes, it is difficult to keep the assumption of the rigid floor and guarantee life safety due to falling cover concrete, glass, finishing parts and so on.

Finally, the design criteria of buildings under earthquake loading have to be decided based on social demand and engineering judgement using the latest seismic engineering of the time by own country. The decision makers have better make use of the existing experience.

## ACKNOWLEDGMENT

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## REFERENCES

NZS3101:2006 *NZS3101:2006 Concrete Design Standards*, Standards New Zealand, Wellington, NZ.

NZS1170.5: 2004 *NZS 1170.5:2004 Structural design actions*, Standards New Zealand, Wellington, NZ.

Examples of Concrete Structural Design to New Zealand Standard NZS3101:2006. Red Book, New Zealand Concrete Society, 2008

Yuji Ishikawa and et al. Seismic Structural Performance of R/C Beam-Column Joints with High-Strength Steel Bars, 8PCEE, December 2007, Singapore

Brendon A Bradley, Misko Cubrinovski, Near-source strong ground motion observed in the 22 2011Christchurch earthquake, Seismological Research Letters, November/December 2011

The Building Center of Japan, The Building Standard Law of Japan on CD-ROM August 2011